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DISCUSSION OF
STRENGTH OF I-BEAMS IN COMBINED
BENDING AND TORSION
(Published in September, 1950)

By Jacob Karol, Melvin W. Jackson, and
Basil Sourochnikoff

STRUCTURAL DIVISION

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DISCUSSION

JACOB KAROL,¹⁴ M. ASCE.—The theory of combined bending and torsion acting on I-beams is presented ably in this paper. In his conclusions, the author states that the main objective of the paper was to analyze stresses rather than to develop practical design formulas. It seems to the writer that an extension of the theory to include the effect of initial lateral bow and initial twist of the beam is all that is needed to make the method applicable to practical design. Moreover, these effects of bow and twist are present even on centrally loaded I-beams, and the designer should have some notion of their importance. This discussion presents the additional theory required to evaluate these effects.

Two additional types of torque curves will be considered: For case 5,

$$T_x = T_o \left[1 - 2 \left(\frac{2x}{l} \right) + \left(\frac{2x}{l} \right)^2 \right] \dots\dots\dots (38a)$$

and, for case 6,

$$T_x = T_o \left[1 - 3 \left(\frac{2x}{l} \right) + 3 \left(\frac{2x}{l} \right)^2 - \left(\frac{2x}{l} \right)^3 \right] \dots\dots\dots (38b)$$

The particular solutions X_1 are: For case 5,

$$X_1 = \frac{T_o l}{2 E_s \kappa} \left\{ \frac{1}{3} \left(\frac{2x}{l} \right)^3 - \left(\frac{2x}{l} \right)^2 + \left(\frac{2x}{l} \right) \left[1 + 2 \left(\frac{2a}{l} \right)^2 \right] \right\} \dots\dots (39a)$$

and, for case 6,

$$X_1 = - \frac{T_o l}{8 E_s \kappa} \left\{ \left(\frac{2x}{l} \right)^4 - 4 \left(\frac{2x}{l} \right)^3 + 6 \left(\frac{2x}{l} \right)^2 \left[1 + 2 \left(\frac{2a}{l} \right)^2 \right] - 4 \left(\frac{2x}{l} \right) \left[1 + 6 \left(\frac{2a}{l} \right)^2 \right] \right\} \dots (39b)$$

After determining the integration constants from the boundary conditions, the values of the angle of twist become: For case 5,

$$\psi = T_o C \left(2 \left(\frac{2a}{l} \right) \left(\cosh \frac{x}{a} - 1 \right) - 2 \left(\frac{2a}{l} \right)^2 \left(\frac{l}{2a} \sinh \frac{l}{2a} + 1 \right) \frac{\sinh \frac{x}{a}}{\cosh \frac{l}{2a}} + \frac{l}{2a} \left\{ \frac{1}{3} \left(\frac{2x}{l} \right)^3 - \left(\frac{2x}{l} \right)^2 + \left(\frac{2x}{l} \right) \left[1 + 2 \left(\frac{2a}{l} \right)^2 \right] \right\} \right) \dots (40a)$$

and, for case 6,

$$\psi = T_o C \left(3 \left(\frac{2a}{l} \right) \left[1 + 2 \left(\frac{2a}{l} \right)^2 \right] \left[\cosh \frac{x}{a} - 1 - \tanh \frac{l}{2a} \sinh \frac{x}{a} \right] - \frac{1}{4} \left(\frac{l}{2a} \right) \left\{ \left(\frac{2x}{l} \right)^4 - 4 \left(\frac{2x}{l} \right)^3 + 6 \left(\frac{2x}{l} \right)^2 \left[1 + 2 \left(\frac{2a}{l} \right)^2 \right] - 4 \left(\frac{2x}{l} \right) \left[1 + 6 \left(\frac{2a}{l} \right)^2 \right] \right\} \right) \dots (40b)$$

NOTE.—This paper by Basil Sourochnikoff was published in September, 1950, as *Proceedings-Separate No. 33*. The numbering of footnotes, illustrations, tables, and equations in this Separate is a continuation of the consecutive numbering used in the original paper.

¹⁴ Kansas City, Mo.

in which $C = \frac{a}{E_s \kappa}$ is the torsional twist constant. The maximum angle of twist at midspan is given by Eq. 11 and the values of R are as follows: For case 5,

$$R = R_{ce} = \frac{l}{2a} \left[\frac{1}{3} + 2 \left(\frac{2a}{l} \right)^2 \operatorname{sech} \frac{l}{2a} - 2 \left(\frac{2a}{l} \right)^3 \tanh \frac{l}{2a} \right] \dots (41a)$$

and, for case 6,

$$R = R_{ue} = \frac{l}{2a} \left\{ 3 \left(\frac{2a}{l} \right)^2 \left[1 + 2 \left(\frac{2a}{l} \right)^2 \right] \left(\operatorname{sech} \frac{l}{2a} - 1 \right) + \frac{1}{4} + 3 \left(\frac{2a}{l} \right)^2 \right\} \dots (41b)$$

The flange stress f_f is found from Eqs. 5 and 40. For case 5,

$$f_f = B T_o \left\{ 2 \left(\frac{2a}{l} \right) \left[1 - \frac{2x}{l} - \cosh \frac{x}{a} + \tanh \frac{l}{2a} \sinh \frac{x}{a} \right] + 2 \left(\frac{2a}{l} \right)^2 \operatorname{sech} \frac{l}{2a} \sinh \frac{x}{a} \right\} \dots (42a)$$

and, for case 6,

$$f_f = 3 B T_o \left(\frac{2a}{l} \right) \left\{ \left[1 + 2 \left(\frac{2a}{l} \right)^2 \right] \left[1 - \cosh \frac{x}{a} - \tanh \frac{l}{2a} \sinh \frac{x}{a} \right] - 2 \left(\frac{2x}{l} \right) + \left(\frac{2x}{l} \right)^2 \right\} \dots (42b)$$

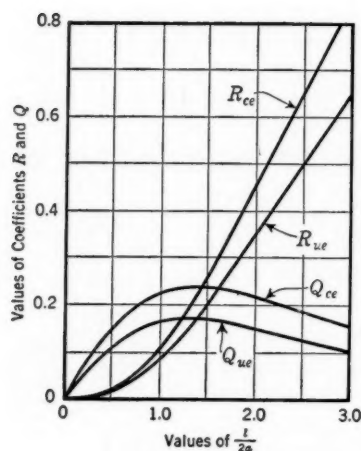


FIG. 7.

in which B is the torsional stress constant, $\frac{ab}{I_y d}$. The maximum flange stress, at midspan, is given by Eq. 14 and the values of Q are: For case 5,

$$Q = Q_{ce} = 2 \left(\frac{2a}{l} \right) \left[\frac{2a}{l} \tanh \frac{l}{2a} - \operatorname{sech} \frac{l}{2a} \right] \dots (43a)$$

and, for case 6,

$$Q = Q_{ue} = 3 \left(\frac{2a}{l} \right) \left\{ 2 \left(\frac{2a}{l} \right)^2 - \left[1 + 2 \left(\frac{2a}{l} \right)^2 \right] \operatorname{sech} \frac{l}{2a} \right\} \dots (43b)$$

The variation of R and Q with $\frac{l}{2a}$ is shown in Fig. 7.

The initial lateral bow is assumed to be parabolic, so that

$$e = e_o \left[2 \left(\frac{2x}{l} \right) - \left(\frac{2x}{l} \right)^2 \right] \dots (44)$$

in which e_o is the bow at the center of the span. For simplicity, the initial twist in the beam is also assumed to be parabolic, hence

$$\phi = \phi_o \left[2 \left(\frac{2x}{l} \right) - \left(\frac{2x}{l} \right)^2 \right] \dots \dots \dots (45)$$

in which ϕ_o is the twist at the center of the beam. For the concentrated load, the added torque due to bow and twist is

$$\Delta T_x = \frac{W}{2} \left[1 - 2 \left(\frac{2x}{l} \right) + \left(\frac{2x}{l} \right)^2 \right] \left[e_o + e_2 \phi_o \right] \dots \dots \dots (46)$$

and corresponds to case 5. For the uniform load, the added torque due to bow and twist is

$$\Delta T_x = \frac{wl}{3} \left[1 - 3 \left(\frac{2x}{l} \right) + 3 \left(\frac{2x}{l} \right)^2 - \left(\frac{2x}{l} \right)^3 \right] \left[e_o + e_2 \phi_o \right] \dots \dots (47)$$

corresponding to case 6. Hence, for the general case, including the effect of eccentricity, bow and twist, the maximum angle of twist at midspan is given by the expressions: For concentrated load—

$$\beta_o = \frac{C R_c \frac{W e_1}{2} + C R_{ce} \frac{W}{2} (e_2 \phi_o + e_o) + \phi_o}{1 - C R_c \frac{W}{2} \left(e_2 + 0.0188 \frac{W l^3}{E I_y} \right)} \dots \dots \dots (48a)$$

and, for uniform load—

$$\beta_o = \frac{C R_u \frac{wl}{2} + C R_{ue} \frac{wl}{3} (e_2 \phi_o + e_o) + \phi_o}{1 - C R_u \frac{wl}{3} \left(e_2 + 0.01075 \frac{wl^4}{E I_y} \right)} \dots \dots \dots (48b)$$

The added term ϕ_o in the numerators of Eqs. 48 accounts for the fact that the beam has an initial twist at the center.

General equations for the flange stress for the torque considered alone are: For concentrated load—

$$f_f = B Q_c \frac{W e_1}{2} + B Q_{ce} \frac{W}{2} (e_2 \phi_o + e_o) \dots \dots \dots (49a)$$

and, for a uniform load—

$$f_f = B Q_u \frac{wl e_1}{2} + B Q_{ue} \frac{wl}{3} (e_2 \phi_o + e_o) \dots \dots \dots (49b)$$

For the general case, and loads equal to n times the design load, the maximum angle of twist is: For concentrated load—

$$\beta_{on} = \frac{C R_c n \frac{W e_1}{2} + C R_{ce} n \frac{W}{2} (e_2 \phi_o + e_o) + \phi_o}{1 - C R_c \frac{n W}{2} \left(e_2 + 0.0188 \right) \frac{n W l^3}{E I_y}} \dots \dots \dots (50a)$$

and, for uniform load—

$$\beta_{on} = \frac{C R_u \frac{n w l e_1}{2} C R_u \frac{n w l}{3} (e_2 \phi_o + e_o) + \phi_o}{1 - C R_p \frac{n w l}{3} \left(e_2 + 0.01075 \frac{n w l^4}{E I_y} \right)} \dots \dots \dots (50b)$$

If the initial bow and twist are zero, Eqs. 50 reduce to Eqs. 31. The stresses for design load are given by Eq. 29 and those for ultimate load are given by Eq. 30. Allowable loads for a given condition are determined by trial using either Eq. 30 or Eq. 32.

The foregoing general theory will now be applied to the author's examples. From the rolling tolerances indicated in the "Pocket Companion" (published by the Carnegie Steel Company), it may be assumed that $e_o = \frac{l}{1,000}$ and $\phi_o = 0.005$. The allowable loads on the 14WF74 beam for various combinations of eccentricity, bow, and twist are shown in Table 3. The detailed calculations are similar to those indicated by the author in his examples.

TABLE 3.—ALLOWABLE LOADS ON 14WF74 BEAM (SPAN, 20 FT)

Case	Deformation	Concentrated load, in pounds		Uniform loading, pounds per foot	
		(3)	(4)	(5)	(6)
(1)	(2)	(3)	(4)	(5)	(6)
	Eccentricity, e_1 in inches.....	0	2	0	2
	Eccentricity, e_2 in inches.....	7	7	7	7
1	Neither bow nor twist.....	37,400	18,700	3,740	1,950
2	Bow, 0.24 in.....	35,200	18,350	3,510	1,920
3	{ Bow, 0.24 in. Twist, 0.005 }.....	34,000	18,100	3,350	1,890

It is worth noting that, for no eccentricity of load, the effect of bow and twist is to reduce the allowable loads as determined by the author by approximately 10%. When the eccentricity of load is 2 in. from the plane of the web, the effect of bow and twist is to reduce the allowable loads by approximately 3%. It is evident then that the effect of bow and twist is large enough to be considered in design.

MELVIN W. JACKSON,¹⁵ Assoc. M. ASCE.—This is a contribution to the solution of a knotty problem which is frequently glossed over in structural design practice because of its complexity. A simply supported beam with torsionally fixed ends, however, is rather uncommon. It is almost impossible to design a connection in riveted construction for this condition. It is also unusual in welded design, for if a beam is torsionally fixed, it is usually fixed against bending.

¹⁵ Asst. Prof. of Civ. Eng., Univ. of Colorado, Boulder, Colo.

The solution studied requires implicitly that the point of application of the load must remain fixed with respect to its original position. Any movement of the point of application of the load due to rotation will alter the solution. Laterally unsupported building spandrels were offered as a common type of problem involving bending and torsion; but where masonry or timber loads are supported by spandrels, there is frequently a shifting or redistribution of load when twisting occurs.

The basis for establishing the allowable normal unit stress in bending needs clarification. The conclusion that

"* * * the allowable stress in bending is affected by the torque and that the primary bending affects the allowable stress for torsional flange bending."

is true, but it is not complete. It should not be implied that Eqs. 32 and 33a can be "conveniently used in design." It is true that, for a beam in bending, the maximum usable stress in flexure is the yield point stress. However, because of lateral instability, a beam may fail in bending below the yield point stress as computed by the flexure formula. If e_1 is zero, Eq. 33a reduces to $f'_b = \frac{f_y}{f_n}$, ignoring this effect of lateral buckling. The current design formula for compression in I-beams for building construction, based on the consideration of lateral instability,¹⁶ is:

$$f_c = \frac{12,000,000}{l d / (b t)} \dots \dots \dots (51)$$

in which f_c is the compression on the extreme fibers of I-beams when $\frac{l d}{b t}$ is greater than 600. Eq. 51 neglects the effect of torsion or accidental eccentricities. Probably both torsion and lateral buckling should be considered in establishing a satisfactory design formula for this problem.

The consideration of principal stress is excluded in the paper. This is consistent with the usual design of I-beams for bending in which the maximum flexural stress is the maximum principal stress. The exclusion of principal stresses does not seem satisfactory, however, if a formula for allowable stress in bending (Eq. 33a) is to be considered. In this case the maximum flexural stress is not the maximum principal stress. Failure of a beam under a long-time load (as distinguished from a load applied by the usual testing machine) will probably start when the maximum principal stress reaches the yield point stress of the material if lateral buckling is not critical. The principal stresses in cases of combined bending and torsion may be substantially larger than the normal stresses.

Much study needs to be done in the solution of structural design problems in combined bending and torsion. The solution for I-beams represents one of the simpler cases. Unsymmetrical spandrel beams supporting eccentric loads probably will continue to be designed for a long time by guess, intuition, and prayer.

¹⁶ "Steel Construction," 5th Ed., American Inst. of Steel Construction, New York, N. Y., 1947, p. 286.

BASIL SOUROCHNIKOFF¹⁷.—The method presented in this paper has been extended by Mr. Karol to include the effect of initial bow and twist. This is an interesting contribution to the theory of torsion and lateral stability of beams.

One can readily agree with Mr. Jackson that much study needs to be done in the solution of problems in combined bending and torsion. Particularly, the determination of combined principal stresses should some day be undertaken. However, the writer finds it hard to agree with the remainder of the views advanced by Mr. Jackson and takes this opportunity to lay particular stress on three important points in which the paper was apparently misunderstood:

1. Contrary to the opinion of Mr. Jackson, the solution does not require that the point of application of the load remain fixed with respect to the original position. Figs. 1 and 6 and the accompanying text clearly show that the sideways movement of the load has been taken into account.

2. If e_1 is zero, Eq. 33a is not reduced to $f'_b = \frac{f_y}{n}$ when the load defined by Eqs. 34a or 34b is reached, since in this case the denominator of the expression for β_{on} becomes infinite. Therefore, the possibility of the beam failing laterally has been taken into account even in the case when no eccentricity is present. An advantage of Eq. 33a over the usual formulas for allowable compressive stress in beams is that it permits taking account of the eccentricity.

3. In the conventional steel framing, the girders carrying moderate torsional loads are connected to columns with light top and bottom angles in addition to the web connections. These angles are designed to be flexible in the longitudinal direction so an undesirable bending moment will not be transmitted to the columns. These angles, however, have enough lateral rigidity to prevent the girder from capsizing due to a torsional load. At the same time, being as a rule connected with only two rivets or bolts each to the girder flanges, they do not introduce an appreciable restraint against bending of the flange in its own plane.

The paper is an approximate analysis of the effect of combined bending and torsion in this widely used system of construction.

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